NUDECK - A New Prestressed Stay-In-Place Concrete Panel for Bridge Decks

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An improved stay-in-place (SIP) precast prestressed concrete panel system is presented. This system eliminates the major drawbacks of conventional stay-in-place precast panels and still maintains their structural and economical efficiencies. The proposed precast panel covers the entire width of a bridge. Therefore, it eliminates the necessity of forming for the overhangs and results in reduction of the time and labor required installing a larger number of individual panels between girder lines. The panel has a full-length gap at the girder lines to maximize the space needed to accommodate the shear connectors of the supporting girders. The precast panel is continuous over the girder with an innovative reinforcement system. Shear keys and reinforced pockets are used in the direction of traffic to provide continuity between panels. Testing shown that the proposed system has superior structural performance to the conventional SIP panel system. All materials used in the production the panel are non-proprietary and readily available. This makes this system cost competitive with that of conventional SIP precast deck system. Details of the system, including design procedure, construction steps and experimental verification, are given. Key words: concrete, prestressed, precast, stay-in-place, bridge, deck.

INTRODUCTION

The great majority of bridges built in the United States have a concrete deck slab. Most of these slabs are cast-in-place (CIP). Many bridge deck construction systems have been developed either for the construction of new bridges or for the rehabilitation of deteriorated bridge decks. Among these systems is the conventional precast stay-in-place (SIP) prestressed concrete deck panel system. This system has been employed successfully in Florida, Texas, Missouri and several other states. The precast SIP deck panel system provides a thin solid precast prestressed concrete panel of 76 to 102 mm (3 to 4 in.) to function as a form for the CIP topping and also to house the positive moment reinforcement. These panels are produced in 1219 to 2438 mm (4 to 8 ft) widths depending on the available transportation and lifting equipment. The precast panels are butted against each other without any continuity between them. They are set on variable thickness bearing strips to allow for elevation adjustment. This system has the advantage of high construction speed compared to the full-depth cast-in-place deck system because of elimination of field forming. However, it suffers from reflective cracks over the transverse joint between the precast SIP panels due to lack of continuity in the longitudinal direction. Also, the prestressing strands are not fully used due to the lack of developed length especially for small girder spacing. Design, detailing, field implementation and test results are available (*1,2,3,4*).

The objective of this paper is to present an improved precast prestressed stay-in-place concrete panel to overcome some the drawbacks of the existing precast SIP panel system. The improved precast stay-in-place deck panel covers the entire width of a bridge. The precast panel is pretensioned from end to end. Each panel acts as a continuous member over the girder in the transverse direction. Transverse and longitudinal continuity is achieved by means of an innovative technique described in the following sections.

DESCRIPTION OF THE SYSTEM

To provide a detailed description of the system, a bridge of 13411-mm (44-ft) width is considered. The deck consists of three 3658-mm (12-ft) spans plus two 1219-mm (4-ft) overhangs. The four supporting steel girders have 305-mm (12-in.) wide top flange. The system consists of a 114-mm (4.5-in.) precast prestressed SIP panel and a cast-in-place concrete topping. The CIP topping thickness can vary from 90 to 144 mm (3.5 in. to 4.5 in.) based on the girder spacing, minimum specified concrete cover and type of live load. Figure 1 shows the cross section of the deck system.

Figure 2 gives a plan view of the SIP precast panel. It covers the entire width of the bridge. The panel length can vary from 1220 to 3658 mm (4 to 12 ft) according to the transportation and lifting equipment available in the field. In this study, the panel length was chosen at 2438 mm (8 ft). At the girder positions there is a full-length gap for a width of 203-mm (8-in.) to accommodate the shear connectors. The gap-width was determined based on having steel girders with one line of shear connectors. However, the width can be increased for the case of using more than one line of shear connectors or using concrete girders. High strength concrete is used to cast the panel. Specified concrete release strength is 27.58 MPa (4.0 ksi) and the specified 28-day compressive strength is 68.95 MPa (10.0 ksi).

The panel is pretensioned from end to end with sixteen, 13-mm (1/2-in.) diameter, low-relaxation, strands of 1,862 MPa (270 ksi). The strands are provided in two layers and uniformly spaced at 12 in. (305 mm) spacing as shown in section A-A in Figure 3.

A minimum clear concrete cover of 1 inch (25 mm) is used for both the top and bottom layers of strands. In order to maintain the gap over the girder positions and to transmit the prestressing force from one part to another over the gaps, when releasing the prestressing force, 28- #19 (#6) reinforcing bars are used in two lay-

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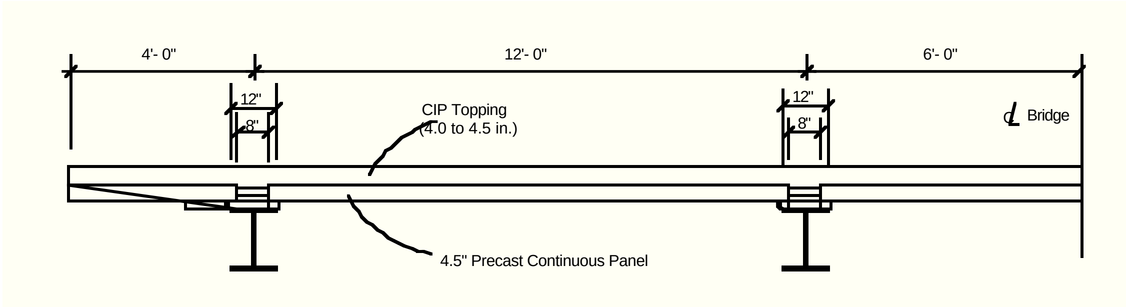

FIGURE 1 Cross-section of the bridge (1.0 ft = 304.8 mm, 1.0 in. = 25.4 mm).

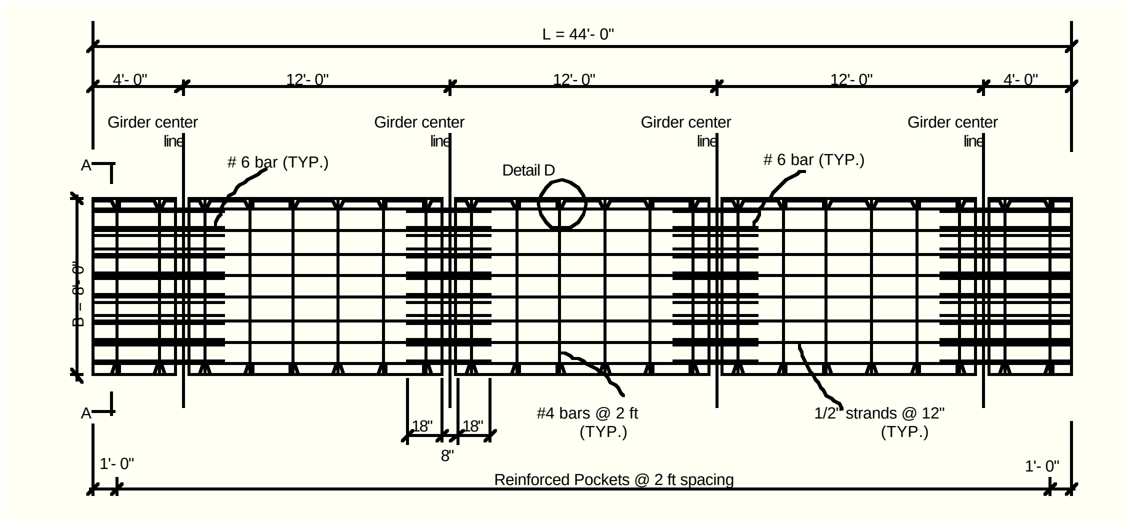

FIGURE 2 Plan view of the SIP panel (1.0 = 304.8 mm, 1.0 in. = 25.4 mm).

ers, as shown in section C-C in Figure 3. These bars have 457-mm (18-in.) embedment length to transmit the compression force from one part of the panel to the next part over the gaps.

To maintain continuity in the longitudinal direction between the adjacent precast panels, shear keys and reinforced pockets are provided. Section 1-1 in Figure 4 shows the dimensions of the proposed shear key. Reinforced pockets are spaced at 610 mm (2 ft) on center. Detail (D) and Section 2-2 in Figure 4 gives the dimensions of the reinforced pocket.

To avoid field forming, a 20-gage 152 mm x152 mm (6 in. x 6 in.) metal sheet is used as a stay-in-place form at the pockets. The panel is reinforced longitudinally with #13 (#4) bars spaced at 610 mm (2 ft). To provide for tension development for the #13 (#4) bars, they were spliced using an innovative confining technique. The splice consists of a loose 229 mm (9-in.) long #13 (#4) bar and a spiral whose size is shown in Figure 5. This technique was separately evaluated (*7*) with small tension specimens and found to produce the full bar yield strength of 414 MPa (60 ksi).

The precast SIP panels are leveled, when set over the supporting girders, using a simple leveling device as shown in Figure 6. The leveling device consists of a 1/2-inch (13-mm) thick plate. The plate has a hole of 22-mm (7/8 in.) diameter. A 19-mm (3/4-in.) nut is welded to the bottom surface of the plate. The plate is mounted between the top flange of the girder and the lower layer of reinforcement of the precast panel. A 178-mm (7-in.) high, 19-mm (3/4-in.) diameter bolt is inserted through the nuts to level the precast panel by it turning up and down. Once the panels are placed over the girders and adjusted with the leveling devices, gaps over the girders are grouted with a flowable mortar mix. The mortar mix should have a compressive strength of 27.58 MPa (4.0 ksi) at time of casting the topping slab. The mortar provides a compression block needed to resist the negative moment over the girders due to loads

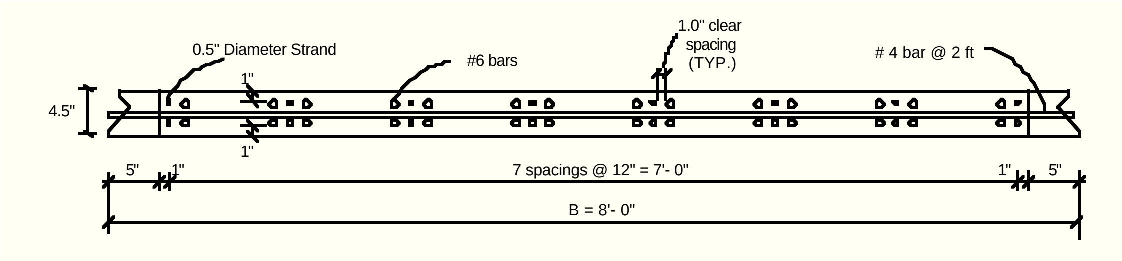

FIGURE 3 Section A-A(1.0 ft = 304.8 mm, 1.0 in. = 25.4 mm).

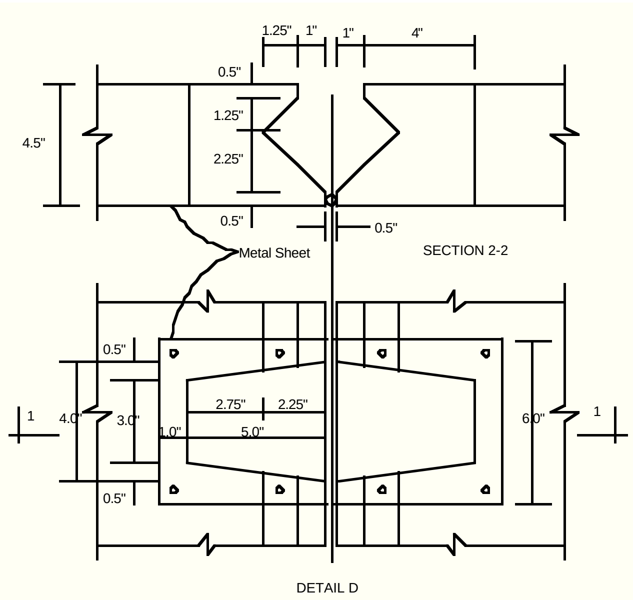

FIGURE 4 Transverse joint details (1.0 ft = 304.8 mm, 1.0 in. = 25.4 mm).

imposed by the concrete paving machine and the self-weight of the concrete topping. It also provides bearing for the precast panel over the girders as shown in Figure 6.

DESIGN PROCEDURE

AASHTO Standard Specifications, 16th Edition, 1997, is used to design the system. The design procedure consists of two different sections: (1) the precast panel (non-composite section); and (2) the composite section. The precast section is designed to support its self-weight, the topping slab self-weight, a construction load of 2.394 kPa (50 lb/ft2) and the loads provided by the concrete paving machine. The composite section is designed to support a superimposed dead loads of 1.197 kPa (25 lb/ft2), barrier self weight, and the live loads. An HS25 design truckload is considered as the live load. For the design of the precast panel, two stages were considered: (1) release of prestress; and (2) casting of the topping slab. At release stage, compatibility and equilibrium equations are applied at the section at the gap to calculate the compressive stress gained in the #19 (#6) bars and the tensile stress lost in the prestressing strands. Thus,

Equation (1):

$$\varepsilon = \frac{A_p f_{pi}}{A_s E_s + A_p E_p}$$

Equation (2):
Compression stress in the reinforcing bars = $\varepsilon (E_s)$

Equation (3):
Tensile stress in the prestressing strands = $f_{pi} - \varepsilon (E_p)$

where: ε = the elastic strain loss in the gap, f_{pi} = tensile stress in the strands just before release, A_s = the cross section area of the reinforcing bars, A_p = the cross section area of the prestressing strands, E_s = the Modulus of Elasticity of the reinforcing bars, E_p = the Modulus of Elasticity in the prestressing strands.

Using Equations (1) to (3) results in ε = 1.164x10^{-3} mm/mm (in./in.), compression stress in the reinforcing bar = 233 MPa (33.76 ksi), and tensile stress in the prestressing strands = 1171 MPa (169.91 ksi). Similar analysis at mid-span between the girder lines needs is conducted to determine the tensile stress in the prestressing strands at that location. The reinforcing bars in the gap must be adequate to satisfy two design criteria: (1) preserve as much prestress in the strands as possible; and (2) transfer that prestress to the adjacent concrete without too much stress concentration. The first criterion was already covered in the preceding paragraph. Satisfaction of the second criterion is not totally clear to the authors. A conservative approach is to use the tension development length as the minimum required embedment into the concrete. The buckling length of the #19 (#6) bars at the gap is also checked to protect these bars from buckling.

At topping slab casting stage, three sections are checked. The first section is the maximum positive moment section between the girders, which is designed as prestressed concrete section. Thus, service concrete stresses and ultimate flexural capacity of the SIP panel are checked. The second and third sections are the negative moment sections at interior and exterior girder lines. These sections are designed as conventionally reinforced concrete sections.

Finally after the CIP topping cures, at service stage, the three sections mentioned previously are checked against superimposed dead and live loads taking into account the composite action.

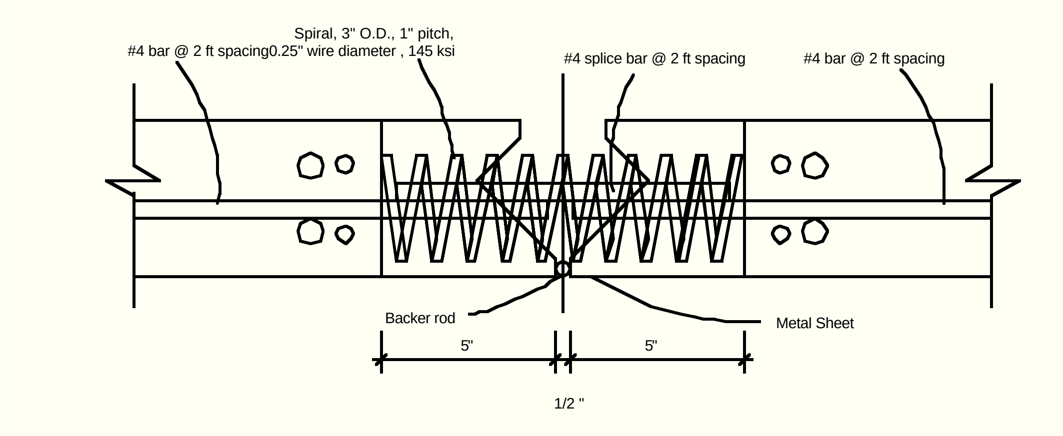

FIGURE 5 Reinforced pocket detail (1.0 ft = 304.8 mm, 1.0 in. = 25.4 mm).

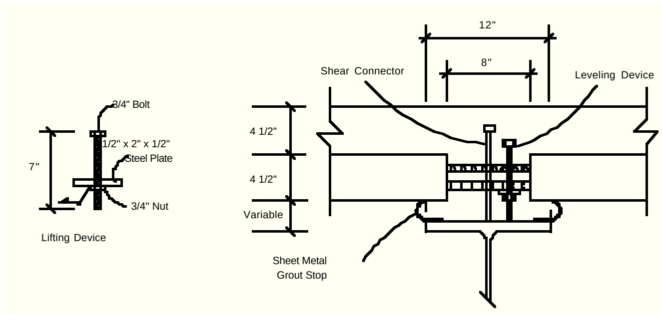

FIGURE 6 Leveling device (1.0 ft = 304.8 mm, 1.0 in. = 25.4 mm).

FIGURE 7 Shear-key and pocket forming.

TESTING PROGRAM

A 6096-mm (20-ft) wide and 2438-mm (8-ft) long bridge was constructed in the structural lab. The bridge consisted of two steel girders spaced at 366 mm (12 ft) and two 122-mm (4.0-ft) overhangs. Two 6096 mm x 1219 mm (20 ft x 4 ft) precast SIP panels were produced in the lab. Wood forming was used to form the shear keys while polystyrene foam was used to form the reinforced pockets, as shown in Figure 7. Wood forming was used to form for the gap over girder lines, as shown in Figure 8. The top surface of the panel was roughened using a silk brush to a height of approximately 13 mm (0.5 in.) to provide for composite action between the precast panel and the CIP topping.

Figure 9 shows details of the panel after forming is completed. Figure 10 shows the stability of the SIP panel during handling. In order to study the behavior of the NUDECK system, two tests were conducted. These are a cyclic load test and an ultimate load test. Figure 11 shows details of the test setup. The test setup simulates two HS25 trucks spaced at 4 ft (1219 mm).

STRUCTURAL BEHAVIOR UNDER CYCLIC LOAD

The cyclic load test was performed up to 2x106 cycles. At 0.7x106 cycles, one hairline crack was noted over each girder line. The number, size, and length of cracks reported in the proposed system were much less than those reported in the conventional SIP panel system, which was tested earlier (*5*,*6*). These cracks closed after removing the load. Strain gages mounted on the top surface of the CIP topping showed that the CIP topping gained some compression stress due to the creep of the SIP precast prestressed panel. This helped to recover the cracks and to minimize their number, size, and length. No reflective cracks over the transverse joints between the SIP panels were noted.

Behavior At Ultimate Load

The test specimen was loaded incrementally until compression crushing in the CIP topping took place at mid-span between the

FIGURE 8 Gap forming.

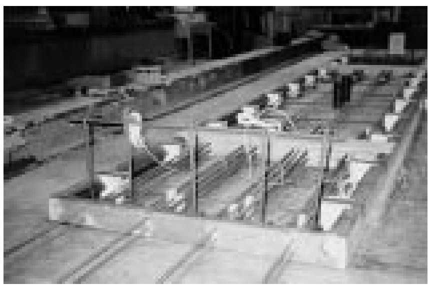

FIGURE 9 Completed forming.

FIGURE 10 Stability during handling.

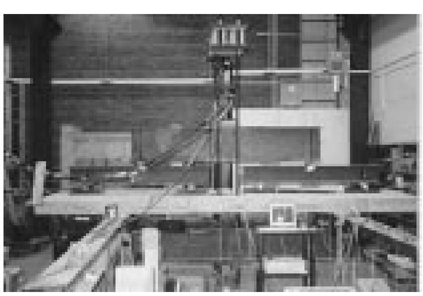

FIGURE 11 Test setup.

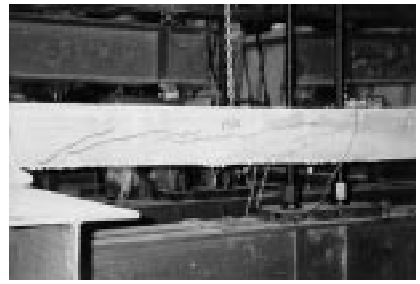

FIGURE 12 Shear cracks at ultimate.

girders. One-way shear cracks were observed in the thin non-reinforced concrete, which was filling the transverse shear key, as shown in Figure 12. However, the shear cracks did not extend beyond the transverse shear key. This was confirmed when the deck was removed for disposal. No sudden failure occurred. After removing the load, the deck returned to its original shape. No residual deflection was noted. This means that the system has a ductile behavior even after failure. Comparing the behavior of this system with the conventional SIP panel system showed that the proposed system has almost double capacity of that of the conventional SIP panel system, it has ductile behavior, and it has less deformation. Up to the failure moment, no reflective cracks were reported and no slippage took place in the splice connecting the longitudinal #13 (#4) bars. Testing program shows that making the proposed SIP panels continuously in the longitudinal and transverse direction leads to better performance, elimination of reflective cracks, and better distribution of live loads.

CONCLUSION

An improved precast stay-in-place deck panel system has been developed. The proposed system has the following advantages compared to the conventional precast SIP deck panel system:

(1) It has higher construction speed because fewer number of pieces need to be handled and field forming of the overhangs is eliminated; (2) The materials used in the production of the panel are non-proprietary and are inexpensive; (3) Elimination of the reflective cracks at the transverse joints; (4) It has superior structural performance under cyclic load; and (5) The system has almost double the capacity of the conventional SIP panel system.

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